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THE COMPACT AERATION ACTIVATED SLUDGE PROCESS AT THE NEWMARKET WATER POLLUTION CONTROL PLANT NEWMARKET, ONTARIO

THE ONTARIO WATER RESOURCES COMMISSION

REPORT ON THE USE OF THE COMPACT AERATION ACTIVATED SLUDGE PROCESS AT THE NEWMARKET WATER POLLUTION CONTROL PLANT NEWMARKET, ONTARIO

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INTRODUCTION

The present design of activated sludge treatment processes for municipal wastes is mainly done on an empirical basis only. The lack of applicable research data and ensuing significant design criteria has resulted in extremely conservative process designs. Ideally, the kinetics of the activated sludge BOD removal mechanism are required for design using the activated sludge process. Depending on the reaction kinetics, axial mixing considerations may or may not be significant.

Over the past decade, a number of studies have been conducted to determine BOD removal efficiencies by the compact aeration activated sludge process. "Compact" aeration is frequently referred to as "short term" or "high rate" aeration. The majority of these investigations have been pilot-scale studies performed in Europe.

This research project was undertaken to determine the feasibility of compact aeration in the activated sludge process treating municipal wastes as encountered in Ontario. The results of such a study, if successful, would be applicable to the design of new treatment facilities and to the extension of existing overloaded plants.

In addition to determining the limitations of the compact

aeration process, other aspects of this system considered were: sludge settling characteristics, sludge production, BOD, phosphate and nitrogen removals and oxygen requirements.

In general, this report is directed towards establishing guide-lines for the design and application of short term aeration systems.

This study was carried out as a joint research program between the Ontario Water Resources Commission and Simon-Carves of Canada, Limited.

THEORETICAL CONSIDERATIONS

There are relatively few conventional design parameters used to design the biological oxidation (aeration) process in the activated sludge system. Traditionally, these have been air supply, retention time and sludge loading ratio (usually described as the ratio of organic load (BOD) to micro-organisms (MLVSS)). Recent additional attention has been focused on oxygen input considerations. The hydraulic retention time has not generally been considered a rigorous criterion in the activated sludge process design. However, usual practice is to provide a retention period of approximately 8 hrs in the aeration section.

The activated sludge process operating at short aeration periods is described as short term or compact aeration. The organic sludge loading remains similar to that used in the conventional activated sludge process. The term "high rate" is sometimes misleadingly used to describe such a system. American usage of the term "high rate" describes processes featuring short retention times, high organic sludge loadings, and partial BOD removals (approximately 60-70%).

Various studies have been carried out to determine the effect of lowering the aeration retention time on the performance

of the activated sludge system. Several investigators, including Okun (1949) and Smith (1952), concluded that the efficiency of BOD removal was primarily a function of the total sludge mass present in the aeration system. Downing and Wheatland (1962) outlined theoretical considerations of the activated sludge process indicating that the greater the concentration of sludge in the aeration units, the more rapidly the waste will be metabolized. Ellis (1965) considered the limitations of the system to be in the ability to return activated sludge at such a rate as to maintain the high concentration of suspended solids required in the mixed liquor.

From the previous discussion, it may be concluded that the BOD removal rate is theoretically dependent on the active sludge mass present in the aeration section. Before the general application of such a concept is made, it must be realized that the organic removal rate is a function of the type of waste being treated, i.e., the relative ease of removal of BOD. Wuhrmann (1954) points out the possibility that some compounds may require a minimum detention time regardless of the solids level in the aeration tank. For sewages containing a significant industrial wastes fraction, pilot-scale studies and/or biodegradability tests would likely be required.

Okun (1949), Pasveer (1954) and Kessener (1935), in their studies on short term aeration, all considered that the rate of organic removal in the activated sludge process was limited by the rate of supply of oxygen to the system. The oxygen transfer limitations imposed by early diffused air systems led to the prolonged aeration periods still generally employed in activated sludge aeration. With the advent of high intensity aeration devices, this restriction of upper limits of organic removal rates was lifted. McNicholas and Tench (1959) indicated that, in their studies, activated sludge flocs had shown a high capacity to resist the dispersion effect of increased agitation resulting from intensified aeration.

High quality effluents have been obtained with relatively high BOD volumetric loadings. Wuhrmann (1954) reported good BOD reductions at loadings up to 170 lb BOD/1,000 cu ft/day. Similarly, v.d. Emde (1963) indicated that high quality effluents could be obtained at loadings up to 200 lb BOD/1,000 cu ft/day. In general, BOD removals of 90 per cent and greater have been readily attained with as little as 2.0 hr aeration. (McNicholas and Tench (1959), Pasveer (1954), Wuhrmann (1954), v.d. Emde (1963) and Benedek (1965)).

EXPERIMENTAL DETAILS

a) Description of Test Facilities

The Newmarket WPCP features a conventional activated sludge treatment system consisting of coarse screening, grit removal, primary sedimentation, aeration, final clarification and chlorination. Two stage anaerobic digestion is used to stabilize raw and waste activated sludge.

The aeration section consists of three tanks each containing four aeration pockets 30 ft x 30 ft x 10.5 ft depth. The total volume of the aeration section is 670,000 gal providing a retention time of 8.0 hr at the design flow of 2.0 mgd. (All flows and volumes are reported in Imperial gallons). The aeration equipment consists of 12 Simcar aerators, each driven by a 7.5 hp motor. Primary effluent may be fed to any aeration cell through 12 in. diameter feed pipes adjacent to each aeration tank.

For the purpose of this compact aeration process study, the last pocket in one of the aeration tanks was isolated by means of a wood partition of 2 in. x 8 in. planks and a plastic liner. This resulted in an aeration test cell, 30 ft x 30 ft x 10.5 ft containing 56,000 gals. The aerator and drive unit originally installed in the test cell were removed and a 7.5 ft standard Simcar aerator driven by a 25 hp motor was then installed.

As control of oxygen input to the aeration cell was considered critical, provision for raising and lowering the aerator was incorporated into the design facility. It should be pointed out that the control system and the raising and lowering mechanism were experimental units only. The control unit consisted of an EIL (Electronic Instruments Limited) Model 49 dissolved oxygen meter, a temperature compensated oxygen electrode, and a proportional control circuit. Aerator power consumption was periodically determined, using a multi-phase kilowatt-hour meter.

As provision was made for step aeration during normal operation, primary effluent was fed to the aeration test cell through the existing inlet pipe and gate valve located in the test bay. For the testing at high flows (1.0 - 1.35 mgd), it was necessary to use two inlet pipes and construct a pipeline from the adjacent aeration tank to the test cell. The mixed liquor was drawn off by the existing adjustable outlet weir and conveyed to the final clarifier (s) used during the various phases of the program. The two final clarifiers (35 ft x 35 ft x 13 ft) provide a maximum cross-section area of 2,400 sq ft corresponding to an overflow rate of 830 gal/sq ft/day and a detention time of 2.3 hr at the design flow of 2.0 mgd. The actual surface area is approximately 1,600 sq ft. This reduced surface area would

become critical if the sludge blanket was near the surface. The clarifiers are equipped with plow type sludge scraper mechanisms. During the early phases of the study, one final clarifier was isolated from the conventional plant and used to clarify mixed liquor from the test cell exclusively. The return sludge well was divided into two sections and return sludge for the short term aeration process was then pumped from this sump, at a maximum rate of 0.7 mgd.

The primary effluent flow to the test section was measured using the existing 3-inch Parshall flume at the head end of the inlet pipe. Initially extreme variations in flow were encountered; these were overcome by installing weir boards in the gates of the other two Parshall flumes, thus providing a relatively constant flow to the test cell. Early in the study, a flow recorder was installed and calibrated against the existing Parshall flume. The instantaneous readings were not more accurate, but the recorder indicated the variations in flow corresponding to the various raw sewage pumps switching on and off. The return activated sludge flow was measured by a Parshall flume prior to entering the test cell.

b) Experimental Program

The research study was divided into four phases. The initial operation of the compact aeration system took place during January, 1968. At this time, the mechanical commissioning of the dissolved oxygen control aeration system was carried out. Due to the low ambient temperatures at this time, it was considered desirable to maintain at least 0.6 mgd flow to the conventional plant to prevent freezing. The mean daily flow available to the compact aeration plant was then a maximum of 0.7 mgd, based on the DWF of 1.3 to 1.4 mgd. As a result, it was decided to operate the test cell initially at flow of 0.4 and 0.7 mgd.

Throughout the study, composite samples of primary effluent were taken daily. In addition, daily grab samples of final effluent, mixed liquor and return sludge were also taken. During each phase, at least two intensive surveys were carried out; grab samples were taken at the above-mentioned locations at hourly intervals over periods ranging from eleven to twenty-four hours. In addition, oxygen utilization rates of the mixed liquor were determined throughout the day.

Ammonia, nitrate and suspended solids determinations were performed in the plant laboratory. The analyses for COD

BOD, filtered BOD, BOD (ATU added) PO_4, solids determinations and total organic nitrogen were carried out at the Commission laboratory in Toronto. The allylthiourea (ATU) modification on the BOD test is performed to inhibit nitrification. (Reference: Montgomery and Borne (1966)). Results thus obtained should represent carbonaceous BOD only. Nitrification suppression in this manner should yield similar BOD results to those obtained from chlorinated effluents as nitrifying organisms do not survive chlorination at chlorine dosages normally applied. Microscopic examinationsof the activated sludge were carried out regularly. Ambient and aeration tank liquid temperatures were measured daily.

PHASE I

During this period, the test cell was intended to operate at a mean daily flow of 0.47 mgd with the dissolved oxygen content maintained between 50 and 60 percent saturation. However, difficulties with the main sewage pump resulted in a regular surge flow ranging from 0 to 0.63 mgd yielding an average daily flow of 0.47 mgd and a nightly flow of 0.16 mgd average.

Mixed liquor suspended solids concentrations of approximately 3,000 to 4,000 mg/l were maintained during this stage. An attempt had been made to increase the MLSS to 6,000 mg/l but severe sludge losses in the final clarifier occurred at solids concentrations above 4,700 mg/l.

PHASE II

On completion of Phase 1, the flow rate and MLSS were slowly increased in an attempt to reach the proposed 0.7 mgd average flow. The sludge settling characteristics at a MLSS of 4,500 mg/l were such that the final clarifier performance became limiting at a flow of 0.55 mgd. Attempts to increase either the flow or the MLSS resulted in sludge losses over the weir of the final clarifier. Severe turbulence in the vicinity of the stilling well was thought to be the cause of the poor performance of the clarifier. This led to the decision to extend the stilling well to a depth of 7 ft (an additional 3 ft). Steady state operation was obtained at a flow of 0.7 mgd and a MLSS of approximately 4,500 mg/l. Again, the dissolved oxygen concentration was kept between 50 and 60 per cent saturation. The detention time in the aeration section during Phase II was 1.9 hr.

PHASE III

For the studies to be carried out at high flow rates (up to 1.35 mgd), further modifications were required to allow both final clarifiers to be used in conjunction with the compact aeration test cell. Prolonged difficulties with the specially installed return sludge pump resulted in a two month delay before the test cell could be put into continuous service; the existing plant return sludge pumps were used for the remainder of the program.

In mid-August, the aeration test cell was put into operation treating the entire sewage flow of approximately 1.1 mgd. This corresponded to a mean detention time of 1.2 hr. The oxygen control limits were set to provide operation at dissolved oxygen concentrations of 20 to 60 per cent saturation. The MLSS concentration was kept at approximately 5,000 mg/l. The results from intensive sampling surveys indicated that the effluent quality was influenced by increasing applied organic load; operation at this detention time (1.2 hr) was considered to be unsatisfactory.

PHASE IV

After four weeks of operation under the conditions of

Phase III, the flow to the test cell was reduced to 1.0 mgd. providing a detention time of 1.35 hr. Again, MLSS were controlled at approximately 5,000 mg/l and the dissolved oxygen concentration varied between 20 and 60 per cent saturation.

EXPERIMENTAL RESULTS

A summary of the analytical results obtained throughout each phase of the study is given in the following tables. Further interpretation and discussion of these results is presented in the section, "Discussion of Results".

In Table I, a summary of the results obtained during Phase I is shown. The range of data represents results determined during the period February 20 to March 5, 1968. Prior to this time, the test unit had been in operation for approximately 4 weeks but the MLSS were considered too high for satisfactory operation. The mean daily flow throughout Phase I was 0.4 mgd. The calculated performance data of Phase I are presented in Table V.

The analytical results from Phase II are summarized in Table II. The data were accumulated from March 18 to April 3, 1968. The average daily flow during Phase II was 0.7 mgd. The performance of the system at this time is shown in Table V.

TABLE I

ANALYTICAL RESULTS FROM PHASE I

February 20 - March 5, 1968
Results in mg/l except as noted

Primary Effluent	Maximum	Minimum	Average
BOD	270	120	180
BOD (Filtered)	130	46	83
COD	416	172	263
NH_3 as N	33	18	27
NO_3 as N	1.2	0.0	0.6
PO_4	58	21	36
SS	215	62	132
Flow (mgd)	-	-	0.4
Aeration Tank Mixed Liquor			
SS	5,100	3,040	3,730
Volatile Content (%)	80	73	77
S. V I. (ml/gm)	269	178	216
Temperature (^{o}c)	7.5	5.0	6.6
Oxygen Utilization (mg O_2/l/hr)	30	18	25
Final Effluent			
BOD	41	14	25
BOD (Filtered)	25	4	9
BOD (ATU)	21	7	13
COD	72	32	48
NH_3 as N	18	1.0	7.8
NO_3 as N	11.0	1.8	6.4
PO_4	31	24	27
SS	11	4	8

TABLE II

ANALYTICAL RESULTS FROM PHASE II

March 18 - April 3 1968
Results in mg/l except as noted

Primary Effluent	Maximum	Minimum	Average
BOD	141	42	102
BOD (Filtered)	69	8	37
COD	230	165	190
NH_3 as N	27	4	10
NO_3 as N	2.8	0.1	1.7
PO_4	32	8	20
SS	245	30	96
Flow	-	-	0.7
Aeration Tank Mixed Liquor			
SS	4,860	3,470	4,180
Volatile Content (%)	69	65	67
S.V.I. (ml/gm)	235	86	133
Temperature (^oC)	9	7	8.1
Oxygen Utilization (mg O_2/l/hr)	30	24	28
Final Effluent			
BOD	21	10	14
BOD (Filtered)	10	4	7
BOD (ATU)	9	4	6
COD	50	22	35
NH_3 as N	10.0	0.0	3.0
NO_3 as N	5.6	2.2	3.6
PO_4	10.6	5.0	8.5
SS	11	4	7

Table III summarizes the analytical data obtained during Phase III. While the data cover the period August 26 to September 18, 1969, operation of the compact aeration system at the average flow rate of 1.1 mgd was continuous from August 12.

In Table IV, a summary of the results from Phase IV is shown. Daily flow to the test cell was controlled at 1.0 mgd and data accumulated over the period September 20 to October 9, 1968.

The calculated performance data for each of the experimental conditions studied are presented in Table V.

TABLE III

ANALYTICAL RESULTS FROM PHASE III

August 26 - September 18, 1968
Results in mg/l except as noted

Primary Effluent	Maximum	Minimum	Average
BOD	260	32	142
BOD (Filtered)	160	11	68
COD	464	91	254
NH_3 as N	32	12	24
NO_3 as N	3.0	0.0	0.5
PO_4	126	12	46
SS	368	42	125
Diurnal flow variation (mgd)	1.35	0.7	1.1
Aeration Tank Mixed Liquor			
SS	7,310	3,480	6,050
Volatile Content (%)	68	42	65
S.V.I. (ml/gm)	157	87	133
Temperature (^{o}C)	20	17	19
Oxygen Utilization (mg O_2/l/hr)	166	60	95
Final Effluent			
BOD	57	15	32
BOD (Filtered)	30	7	15
BOD (ATU)	22	5	14
COD	95	24	56
NH_3 as N	1.2	0.0	0.2
NO_3 as N	5.0	0.2	1.9
PO_4	47	9	24
SS	74	5	23

TABLE IV

ANALYTICAL RESULTS FROM PHASE IV

September 20 - October 9, 1968
Results in mg/l except as noted

Primary Effluent	Maximum	Minimum	Average
BOD	226	162	192
BOD (Filtered)	130	84	105
COD	502	323	392
NH_3 as N	38	32	35
NO_3 as N	0.2	0.0	0.1
PO_4	74	44	54
SS	155	90	117
Flow (mgd)	-	-	1.0
Aeration Tank Mixed Liquor			
SS	5,320	5,116	5,200
Volatile content (%)	68	68	68
S.V I.(ml/gm)	173	107	145
Temperature (°c)	19	17	18
Oxygen Utilization (mg O2/l/hr)	154	90	110
Final Effluent			
BOD	46	15	28
BOD (Filtered)	16	4	9
BOD (ATU)	24	11	15
COD	83	37	56
NH_3 as N	19	9	12
NO_3 as N	2.5	0.5	1.9
PO_4	30	25	28
SS	16	8	11

TABLE V

CALCULATED PERFORMANCE DATA - COMPACT AERATION SYSTEM

(Using Primary Effluent as input)

Phase Date (1968)	Volumetric BOD Loading	Organic Load * Ratio (F:M)	REMOVAL BOD	BOD** (CARB)	SS	NH_3	PO_4	COD	Sludge Production	Oxygen Utilization	Retention Time in Aeration Section	Clarifier Surface Overflow Rate
	lb BOD/day/ 1,000 ft^3	lb BOD/day/ lb MLSS	%	%	%	%	%	%	lb SS produced/ lb BOD removed	lb O_2/lb BOD removed	hr	gal/day/ft^2
Phase I Feb. 20 - March 5	80	0.34	86	93	94	71	25	82	-	0.55	3.4	333
Phase II March 18 - April 3	80	0.31	86	94	93	70	57	82	-	0.61	1.9	580
Phase III Aug. 26 - Sept. 18	173	0.47	78	90	82	99	48	78	1.29	1.05	1.2	510
Phase IV Sept. 20 - Oct. 9	214	0.65	85	92	91	66	48	86	0.79	0.91	1.35	460

* Based on aeration volume only
** Carbonaceous

DISCUSSION OF RESULTS

a) Waste Characteristics

From the data in Tables I through IV, it can be seen that the primary effluent BOD ranged from 102 mg/l to 192 mg/l. Slightly less than 50% of this BOD was in the soluble and colloidal form. This variation in primary effluent strength resulted in Phases I and II operating under identical organic loadings, although the hydraulic loadings were considerably different. Several BOD determinations on primary effluent were performed using allylthiourea (ATU) as an inhibitor of nitrification. These ATU-BOD results averaged 90% of the standard BOD results indicating little nitrification of primary effluent taking place during the BOD incubation. This minimal nitrification has been attributed to insufficient nitrifying organisms being present in raw and settled sewage.

The raw sewage received at the treatment plant was not solely domestic waste. The Town of Newmarket has some light industry discharging industrial wastes to the municipal treatment plant. Periodic discharges of plating wastes have been observed and there is some question as to whether the relatively poor settling sludge is attributable to such wastes.

b) <u>Compact Aeration System</u>

The compact aeration process performed adequately throughout the course of the programme. During the initial operation of the system, many minor mechanical problems were encountered, e.g. failure of aerator level control unit, limit switch failure, aerator drive coupling sheared, and difficulties with the return sludge pumping system. The dissolved oxygen controller proved to be extremely reliable, but it should be pointed out that, with such a controller, the control system is only as reliable as the dissolved oxygen probe in use. Previous difficulties with dissolved oxygen meters have invariably centered around the sensing probe. The electrodes used in the study had a working life of approximately 8 weeks

Throughout the study, the temperature of the aeration tank contents showed a seasonal variation but were otherwise unaffected by the ambient temperatures. For example, the range of liquid temperatures during winter operation was $5^{o}C$ to $7.5^{o}C$ while the ambient temperature ranged from $-23^{o}c$ to $0^{o}C$. Liquid temperature during summer operation varied from $17^{o}C$ to $20^{o}C$.

Rather poor mixed liquor settling characteristics were encountered during all phases of the programme. Although the absolute S.V.I. values lose some significance when considering systems having high suspended solids concentrations, in this study the settled sludge usually occupied some 75 to 80% of the total cylinder volume (based on 30 min. settling test). This relatively poor settling resulted in a high sludge blanket in the final clarifier. As previously stated, the markedly reduced surface area coupled with the poor settling sludge gave rise to occasional settling problems in the final clarifier. The short term aeration studies reported in the literature indicated that good sludge settling was usually obtained including operation at solids concentrations in excess of 10,000 mg/l. Periodic microscopic examination of the activated sludge did not indicate the presence of any filamentous organisms in sufficient quantities to induce poor settling. The protozoa constantly present during all the tests were of the Aspidiscus and Vorticella species, also observed were protozoa of the Lionotus, Opercularia, Carchesium and Epistylis species. Rotifers and nematodes were seen occasionally.

c) Effluent Quality

The final effluent characteristics as presented in Tables I - IV are averages of results obtained during the corresponding study periods. While these values indicate overall performance, detailed interpretation of these data should be made cautiously. As previously stated, intensive surveys were carried out during each phase. Through such surveys, diurnal variation in the performance of the system was observed.

The ATU-BOD values shown in Tables I - IV represent the carbonaceous BOD in the effluent. These results would compare to those obtained from activated sludge effluents after chlorination.

During Phase I, the intensive studies indicated that relatively little change in effluent quality occurred over the period 10:00 am to 8:00 pm even though the primary effluent BOD increased from 100 mg/l to 200 mg/l during the day.

An assessment of the operation of the system in Phase II was limited to some extent by the seasonal thaw that resulted in high volumes of relatively weak strength

sewage entering the plant. The intensive survey carried out at 0.7 mgd flow revealed little change in primary effluent BOD concentration and also little change in effluent quality throughout the day. As the applied organic loading was the same as in Phase I, such a result was not unexpected.

A significant diurnal variation in effluent quality was observed in the performance of the system in Phase III. In Figure I, the change in effluent quality through the course of the day is shown. The BOD and COD curves reflect the effect of increased loading during the day. The deteriorating quality is difficult to explain. The oxygen utilization rates remained high (up to 165 mg O_2/l/hr) during the day and the dissolved oxygen concentration did not fall below 2.0 mg/l. Apparently the assimilative capacity of the process was exceeded. During the day, the volumetric BOD loading was as great as 375 lb BOD/1,000 cu ft/day. The filtered mixed liquor BOD's closely followed the effluent BOD indicating that the poor effluent quality was not a result of subsequent organic release in the final clarifiers.

The limit for satisfactory operation of the compact aeration process appears to have been reached in Phase IV. In Figure 2, the variation in effluent quality throughout the day

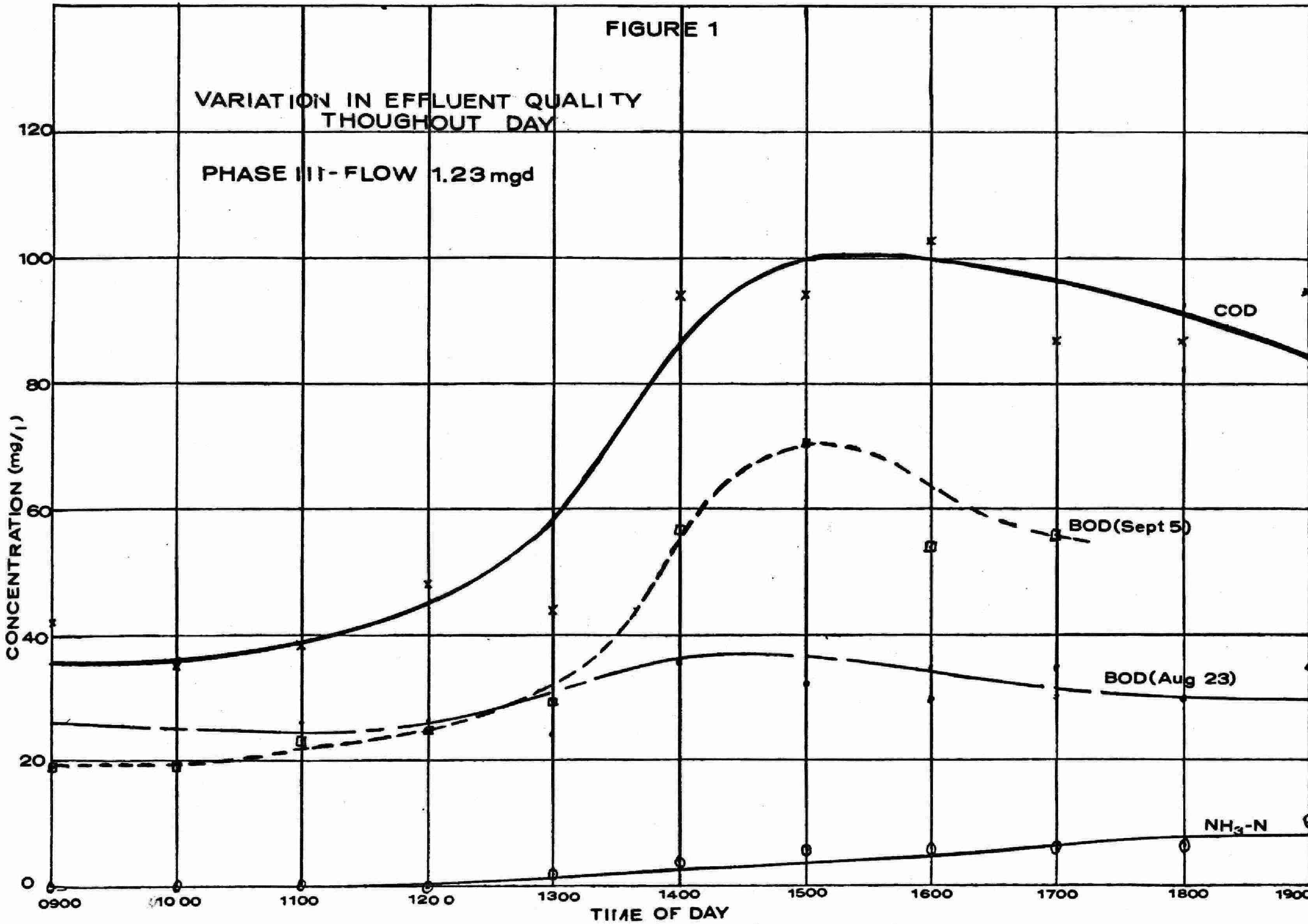
FIGURE 1
VARIATION IN EFFLUENT QUALITY
THOUGHOUT DAY
PHASE III - FLOW 1.23 mgd
COD
BOD(Sept 5)
BOD(Aug 23)
NH3-N
CONCENTRATION (mg/l)
120
100
80
60
40
20
0
0900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
TIME OF DAY

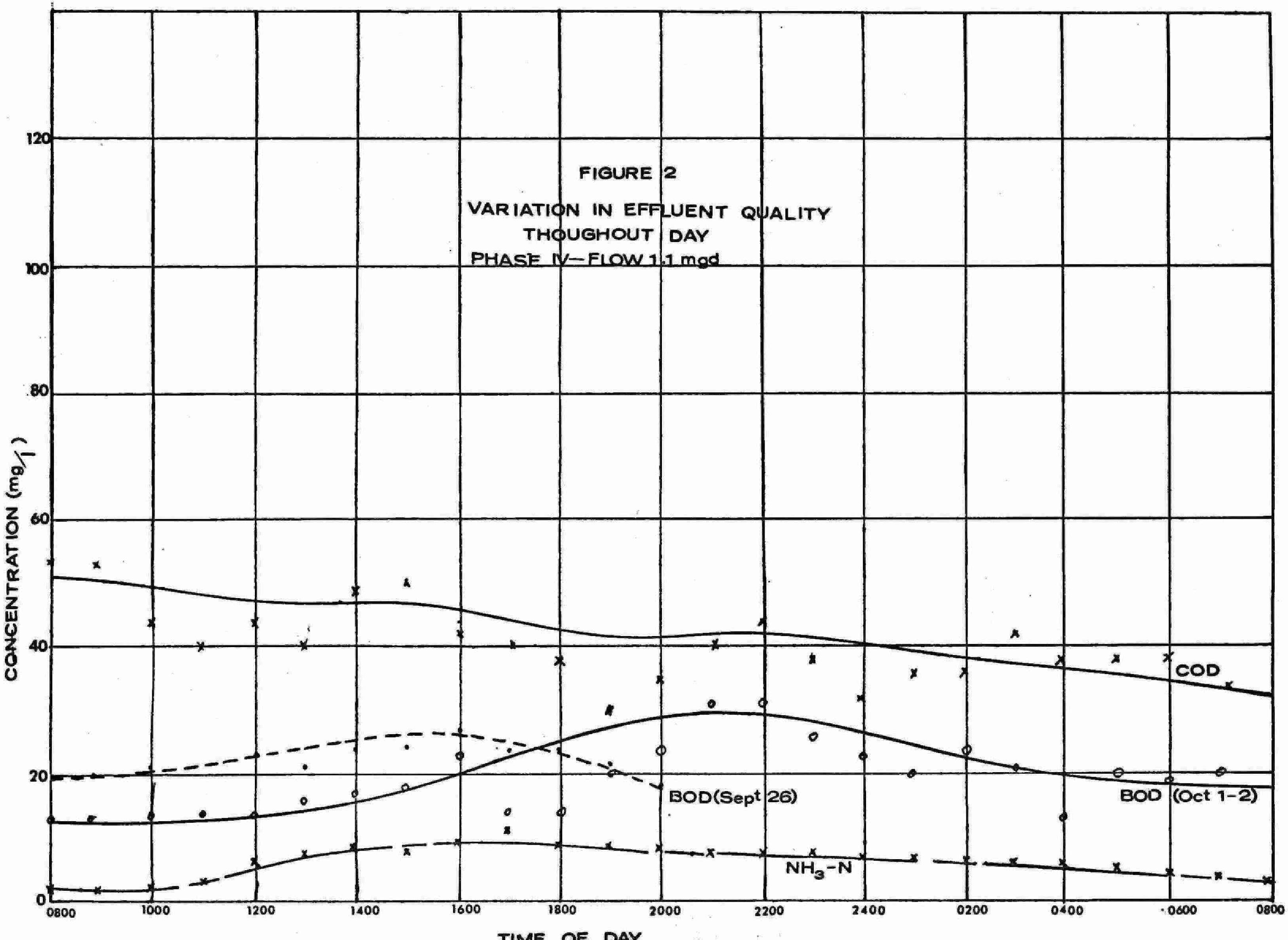

FIGURE 2
VARIATION IN EFFLUENT QUALITY
THOUGHOUT DAY
PHASE IV—FLOW 1.1 mgd

is plotted. The slight increases in BOD are likely the result of increasing applied organic load. Extreme deterioration of effluent quality was not observed.

The diurnal variation in effluent quality as exhibited in Figures 1 and 2 indicates that analysis of a particular process system can be a rather complex procedure. Frequently systems are claimed to effect complete oxidation, nutrient removal, etc.; a thorough evaluation of the system is required to substantiate such claims.

d) Calculated Performance Data

The calculated performance data for each phase of the program are presented in Table V.

The volumetric BOD loading is expressed in terms of daily organic load applied per thousand cubic feet of aeration tank volume. Presently, a figure of approximately 30 lb BOD/day/1,000 cu ft is used in sizing activated sludge process aeration tanks.

The use of a fixed volumetric organic loading parameter for design appears to be extremely limited. Such a concept does not individually assess such factors as oxygenation capacity, waste strength, waste characteristics, mixed

liquor solids levels and retention time; present technology allows several of these factors to be varied independently with satisfactory performance still attained.

The loadings in Phases I and II are identical as a result of the weak sewage that the plant received during Phase II, a period of spring thaw. The loading presented for Phase III is based on the average daily flow. However, during the day, the instantaneous loadings regularly exceeded 300 lb BOD/ 1,000 cu ft/day. In Phase IV, the maximum loadings encountered during the day were approximately 250 lb BOD/1,000 cu ft.day. Based on average daily flows, an acceptable volumetric organic loading limit would appear to be in the order of 220 lb BOD/ 1,000 cu ft/day.

The organic load ratio represents the activated sludge loading factor, i.e. daily organic load applied divided by mass of activated sludge solids in the system. This organic load factor is frequently described as the food to microorganism (F:M) ratio. In Table V, the F:M ratios shown are calculated using the activated sludge solids in the aeration section only. These figures are somewhat misleading, as the mass of solids in the system is considerably greater than that in the aeration tank only. In conventional plants, some 10 to 15% of the active sludge

mass is located in the sludge blanket in the final clarifier. With this compact aeration system, up to 70% of the active sludge mass was held in the final clarifiers. Unless no storage of activated sludge takes place in the final clarifiers, some revision in the method of calculating organic load ratios is required. Considerable data will have to be accumulated from compact aeration systems, before valid estimates of the active sludge mass in the system can be made. Conventional activated sludge plants operate at a loading ratio of approximately 0.30 lb BOD/day/lb MLSS.

The per cent BOD removal obtained in Phases I, II and IV is remarkably consistent considering that the primary effluent BOD was 102 mg/l in Phase II and 192 mg/l in Phase IV. The relatively poor performance in Phase III was likely the result of excessive hydraulic flows being applied to the aeration section during the periods of peak daily flow.

If the limit of the compact aeration system is considered to be operating as in Phase IV, further investigations will likely be required to confirm that such performance can be attained under winter conditions, i.e. aeration tank temperature 5^{o}C. Present conventional activated sludge systems show no adverse effect of low temperature operation; this is

likely the result of gross overdesign.

With the exception of occasional sludge "bulking", suspended solids removals remained satisfactory throughout the program.

Although the settling tests indicated a relatively poor settling sludge, satisfactory clarification was generally achieved. This apparent discrepancy may be attributed to the acknowledged lack of correlation between settling tests and performance of clarifiers.

With considerable interest currently being shown in nutrient discharges to receiving waters, some assessment of the nutrient removal capabilities of the compact aeration process is required. Table V indicates that approximately 70% of the NH_3 - nitrogen was removed except during Phase III, when extremely high nitrogen removals were effected. Unfortunately, no reasonable explanation that could readily be authenticated can be given for the high ammonia removals obtained in Phase III. In an attempt to determine where the removed nitrogen was going, some semblance of a nitrogen balance was attempted. These data are presented in Table VI. The column headed "Deficit" indicates the percentage of nitrogen removed that is not accounted for by nitrate production or sludge production (12% nitrogen content in sludge is assumed). It can be seen

TABLE VI

AMMONIA - NITROGEN REMOVAL

	NH_3-N Load lb/day	Removal %	Removal lb/day	NO_3-N Production lb/day	N in Sludge at 12% N Content lb/day	Deficit %
Phase I	108	71	77	23	17	48
Phase II	70	70	49	13	19	35
Phase III	295	99	292	17	187	30
Phase IV	385	66	254	20	155	31
Phase IV Intensive 24 hr day	275	72	198	15	155	14

that approximately one third of the ammonia nitrogen removed is unaccounted for. Even a 24 hour intensive study resulted in 14% of the ammonia nitrogen removed unaccounted for. Possible nitrogen removal mechanisms that could explain the deficit are ammonia stripping in the aeration section and denitrification to nitrogen gas in the final clarifiers. Nitrate analyses on filtered mixed liquor samples yielded no conclusive evidence of denitrification taking place. Throughout the study, nitrification and possible denitrification were random occurrences. At this stage, it would appear that a practical design to effect nitrification and denitrification is not feasible.

Phosphate removal throughout all phases of the program was appreciable. However, the effluent phosphate concentrations

reflected the applied phosphate load. The removals of approximately 50% are somewhat higher than the 25% removal figure commonly quoted for conventional activated sludge plants. Adsorption rather than removal through assimilation and subsequent sludge production may account for the higher phosphate removals.

COD reductions, per se, have little significance at this time. However, the use of a conserved parameter (relatively time independent) is required to adequately assess effluent quality. The results of this particular study indicate that a COD value of 50 mg/l or less is indicative of a quality effluent. Some well-defined analytical parameter such as COD, TOD or organic carbon should supplant the ill-defined, poorly reproducible, non-conserved BOD determination.

The sludge production figures presented in Table V require considerable explanation. In Phases I and II, an accurate estimate of sludge production could not be attempted for the following reasons:

i) a quantity of activated sludge solids leaked into the upstream portion of the aeration tank containing the test cell. Approximately 5,000 lb of solids were lost in this manner. With the relatively low mass of BOD

removed and low sludge production, the solids lost upstream were a significant fraction of the total sludge production.

ii) Periodic upsets resulted in the loss of sludge over the weirs of the final clarifier.

iii) Some additional sludge loss took place through leakage in the improvised return sludge well.

In Phase III and IV, a reasonable estimate of sludge production could be made. The previous errors outlined were minimal or non-existent. The only problems encountered with accurate measurement of sludge production were:

i) The valve used for continuous wasting of sludge would occasionally plug, and

ii) In Phase III the plant received a heavy load of clay resulting in very high return sludge concentrations (12,000 - 13,000 mg/l) of low volatile content.

Actual sludge production was probably some 20% lower than the figures presented in Table V.

The oxygen utilization figures in Table V were calculated from the average of oxygen utilization rates made during each Phase. Actual oxygen consumption per lb BOD removed would be slightly lower as most of the uptake rates were made during the

day, a period of higher organic load and consequently higher oxygen utilization rates.

e) <u>Design Considerations</u>

The results of this study indicate that the short term aeration process may readily be used in the activated sludge treatment of municipal wastes. A minimum liquid retention period of 1.35 hr is required in the aeration section. Satisfactory operation can be attained at volumetric BOD loadings of up to 220 lb BOD/1,000 cu ft/day. In the actual design of a compact aeration system, it would appear that the 1.35 hr retention period should be applied to the maximum plant flow, i.e. usually 3 x DWF. This would result in a nominal retention period of 4 hr based on DWF. The oxygenation capacity required should be determined on the basis of organic load removed. Further work is required to determine whether variable oxygenation capacity is required. A clarifier surface overflow rate of 500 gpd/sq ft should provide satisfactory clarification for the compact aeration process. Ideally, provision would be made to eliminate extreme hydraulic surges. This could be through the use of variable speed pumps or storm flow storage for combined systems. Return sludge pumping facilities of 100% of maximum DWF are required. It must be pointed out that the compact aeration process requires a more

competent operator than those usually responsible for the operation of small treatment plants.

f) Limitations of Compact Aeration Process

Previous limitations on the activated sludge process have been partially due to limited oxygen transfer capabilities.. This study indicates that the minimum retention period is in the order of 1.35 hr; aeration equipment that is capable of meeting the high oxygen requirements associated with these short aeration periods is presently available. Further work is required to investigate the feasibility of higher organic sludge loadings while still using the short aeration periods common to the compact aeration process.

With the limitations of the aeration equipment overcome, the difficulties with advances in activated sludge treatment lie in a materials transport problem. As the upper limit of gravity thickening of activated sludge is 1½ to 2%, the problem lies with solids separation of high MLSS suspensions. With present clarifiers, the major difficulty appears to be a solids loading problem rather than a hydraulic one.

CONCLUSIONS

1. Satisfactory performance of the compact aeration activated sludge process was obtained at volumetric organic loadings of 220 lb BOD/1,000 cu ft/day and retention periods as low as 1.35 hr.
2. The compact aeration process is capable of producing an effluent meeting Commission objectives in BOD and suspended solids.
3. With the exception of the study at 1.2 hr retention, ammonia nitrogen removals were constant at 70% removal for retention times varying from 1.35 to 3.4 hr.
4. Phosphate removals were inconsistent ranging from 25% to 57%.

RECOMMENDATIONS

1. The compact aeration process has been proven feasible and it is recommended that consideration be given to this process for future installations.
2. An immediate application of this process is the modification of existing waste treatment systems subjected to overloading (hydraulic or organic).
3. Consideration should also be given to the application of this process in cases where wide variation in influent quantity and quality is expected during the year. In such cases the system would operate as a compact process during periods of high loading and as a conventional process during low load periods

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